GROUND WATER ASSOCIATES, INC.
16 Commercial Drive, P.O. Box 280, Dracut, Massachusetts 01826 (508) 970-5388

WELL YIELD TEST REPORT

ORLEANS, VERMONT

TARBOX MEADOW WELL SITE

CONDUCTED FOR:

WRIGHT ENGINEERING, LTD

RUTLAND, VERMONT

BY:

GROUND WATER ASSOCIATES, INC.

DRACUT, MASSACHUSETTS

MARCH, 1989

GROUND WATER ASSOCIATES, INC.

WELL YIELD TEST REPORT

ORLEANS, VERMONT

TARBOX MEADOW WELL SITE

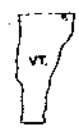
INTRODUCTION

Hydro Group, Inc. was contracted by Wright Engineering, LTD to install an 8-inch diameter test well and several observation wells at the Tarbox site located near the Willoughby River in Orleans, Vermont (Figure 1). Aquifer testing was then conducted by pumping the 8-inch test well, and monitoring several 2.5-inch diameter observation wells in the vicinity. The 2.5-inch diameter observation wells and the 8-inch diameter test well were installed between November and December, 1988. Aquifer evaluation consisted of performing a step-rate pumping test, followed by a long-term, constant-rate pumping test, and subsequent analysis of aquifer and pumping well hydraulic characteristics. The step-rate test was conducted on December 9, and the constant-rate test was conducted for a period of 72 hours between December 12 and 15, 1988. Aquifer recovery measurements were made for 24 hours following the conclusion of the pumping period. Samples of well discharge were collected by Wright Engineering prior to the conclusion of the pumping period for laboratory analyses to assess ground water quality at the site.

In addition to the observation wells, three 2.5-inch diameter monitoring wells were installed near three residential underground fuel oil storage tanks to assess potential ground-water contamination from the tanks.

The following report is intended to meet the objectives of the State of Vermont Department of Health Services Well Yield

Figure 1. Project Location Map.
Adapted from the Memphremagog Quadrangle
U. S. Geologic Survey 15 minute series
topographic map, 1953.



QUADRANGLE LOCATION

0 5280 feet

Test Report as described in the state's ground-water source development guidelines. The report includes:

- Test well installation details,
- Description of area hydrogeology,
- Description of aquifer test methodologies and analyses,
- Definition of aquifer characteristics,
- Safe, sustained yield for the well site,
- Identification of potential sources of aquifer contamination,
- Delineation of wellhead protection areas, and
- Assessment of water quality.

TEST WELL INSTALLATION

2.5-Inch Diameter Wells

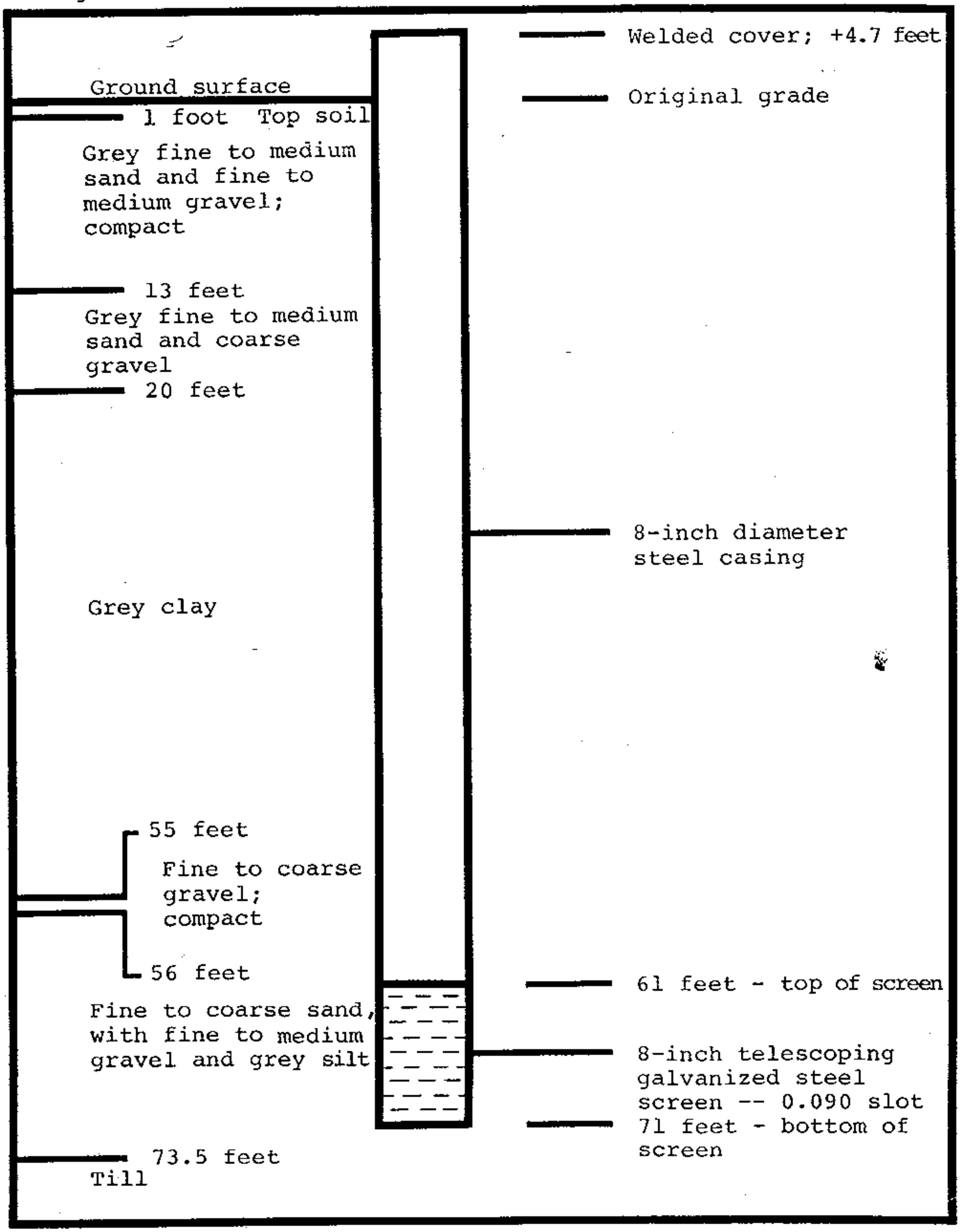
A test drilling survey at the Tarbox site was conducted in November, 1977 by Hydro Group, Inc., to locate optimal aquifer formations. Additional 2.5-inch diameter wells were installed at the site in November and December of 1988 to serve as observation wells during the aquifer tests. The location of 2.5-inch diameter wells used in this aquifer evaluation are shown in Figure 2. Boring logs are provided in Appendix A. The 2.5-inch wells were drilled using the drive and wash boring method.

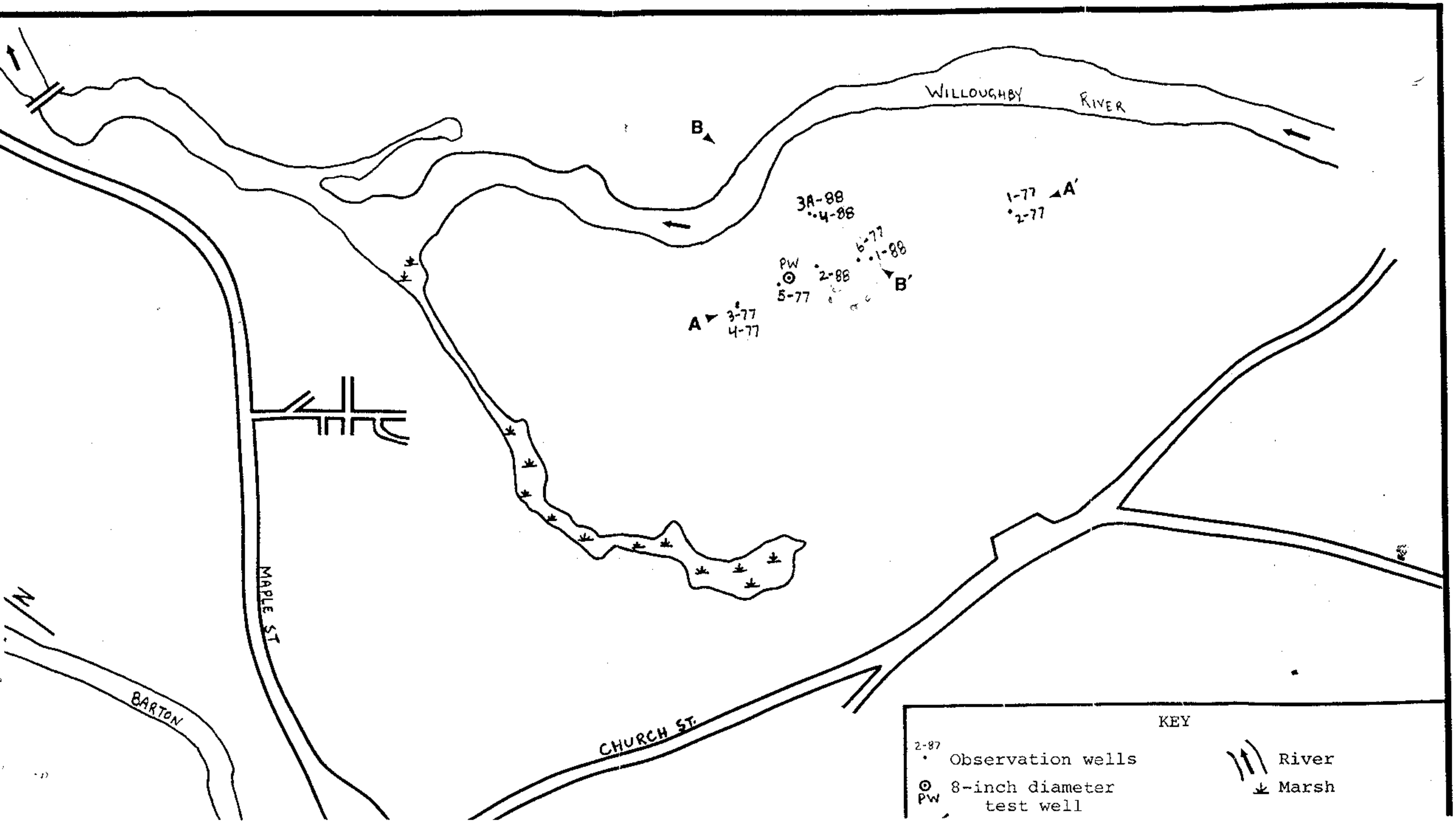
Monitoring wells were installed at residences on Church Street, East Street, and at the intersection of School and Union Streets in Orleans. A polyvinyl chloride (PVC) screen (1.5-inch diameter) was exposed at the bottom of each of the borings, and a flush-threaded solid PVC riser extends to the ground surface. The monitoring wells were completed with bentonite and cement seals, and are protected by 2.5-inch diameter pipes.

8-Inch Diameter Test Well

The 8-inch diameter naturally-developed well was constructed using a cable-tool drilling rig. Figure 3 shows the production well construction plan and geologic log. The production well penetrated 73.5 feet of unconsolidated material. The bottom 17.5 feet of the formation consists of fine to medium gravel with some gray silt. A 10-foot, 90-slot, galvanized screen was

Figure 3. Well Construction Diagram





WILLOUGHBY RIVER
B
B'
A
A'
1-77
2-77
3A-88
4-88
6-77
1-88
PW
2-88
5-77
3-77
4-77
MAPLE ST
BARTON
CHURCH ST.
N
KEY
2-87 Observation wells
PW 8-inch diameter test well
River
Marsh

installed between depths of 71 and 61 feet. The well was developed to a sediment-free condition using a double-surge block agitator and suction lift pumping.

HYDROGEOLOGY

Willoughby River Watershed

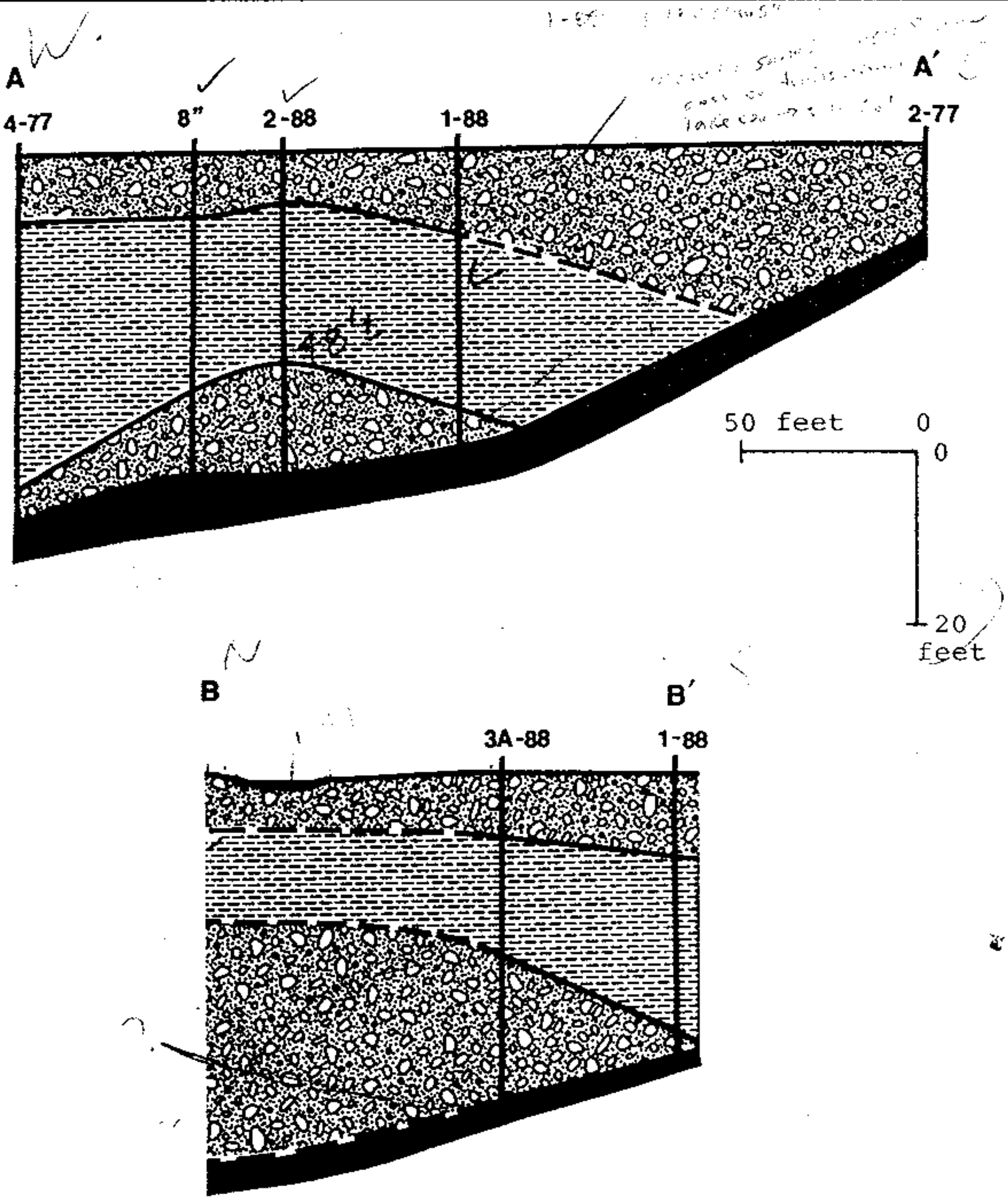
The Willoughby River watershed from the well site in Orleans comprises over 55 square miles in area, and includes the drainage for Lake Willoughby to the southeast. Relief in the watershed is greater than 2,200 feet. Using a conservative estimate of 6 inches for annual infiltration from precipitation (Vermont Department of Health, 1988) 11,000 gallons per minute (gpm) of total recharge is available for withdrawal from the watershed. Watershed area and recharge calculations can be found in Appendix B.

Geology

Structurally, the Tarbox site lies in the Barton River formation (Doll, 1951). The formation consists of "intercalated impure calcareous rocks, ranging from limy quartzites to limestones, amphibolitic layers, slates, phyllites, quartzites, and schists." Although no drilling proceeded into bedrock, Doll (1951) has observed outcrops nearby in Orleans to be "dominantly limestone". The Willoughby River flows roughly north-west through the Tarbox site. Immediately east of the site are the Willoughby Falls, where the river cascades over exposed bedrock, and below which the river meanders in a floodplain. The floodplain is nearly 800 feet in width at the well site. The hillside which surrounds the site (on the south side of the Willoughby River) appears to be bedrock covered by a thin soil layer.

The overburden (unconsolidated) materials at this site are characterized based on Hydro Group's test well logs (Appendix A). A geologic log of the production well is included as part of Figure 3. Generally, the site is characterized by glacial till which overlies the bedrock surface, and which was encountered at depths between 29 and 85 feet at this site. Above the till, a layer of coarse-grained sand and gravel (5 to 30 feet thick), deposited as glacial outwash, is overlain by a thick (30 to 45 feet thick), finer-grained stratum composed primarily of gray clay. The clay unit is thought to have been deposited in a glaciolacustrine environment of quiet, deep waters. The Surficial Geologic Map of Vermont (Doll, 1970) indicates that this site was covered by a glacial or post-glacial lake dammed by the Burlington ice. The upper 10 to 22 feet of unconsolidated materials represent recent alluvial deposits of sand, gravel, cobbles, and boulders. Test well logs indicate a thin (one foot thick) topsoil is present throughout the site.

Figure 4 shows schematic geologic cross-sections of the Willoughby River valley at the Tarbox site based on test well logs. Insufficient data are available to determine the lateral extent of the overburden units described above. The well logs indicate that the overburden deposits thin toward the bedrock valley walls observed to the south, east, and west of the site. It is likely that similar geologic conditions exist to the north of the site, and across the Willoughby River floodplain in this area.



Sands and gravels, recent alluvium

Silts and clays, glacio-lacustrine

Sands and gravels, glacial outwash

Till

Figure 4. Schematic cross section of subsurface geology, Orleans, Vermont.

Geologic materials encountered during monitoring well installation (topographically above the floodplain) generally consist of sand and gravel to depths of 12 to 22 feet, below which lies gray clay to observed depths between 12 and 24 feet. The existence of the clay at these higher elevations indicates that this area was also covered by a glacial lake.

Ground Water Flow

The upper, alluvial deposits and the lower, glacial outwash (sand and gravel) deposits serve as conduits for ground-water flow. These water-bearing units are referred to as aquifers. The glaciolacustrine clay layer impedes ground-water flow, and is considered to be an aquitard. Ground-water recharge in the contributing watershed is derived from infiltrating precipitation. Because the lateral extent of geologic units (particularly the confining clay) are not verified, exact recharge zones to the lower aquifer cannot be identified. It is likely, however that recharge to the lower aquifer is provided from bedrock highlands beyond the lateral extent of the low-permeability, confining clay layer. Some of the test wells screened in the lower aquifer exhibit flowing artesian conditions, indicating that upward ground-water flow gradients exist at the site. The artesian conditions give further support to the theory that aquifer recharge is largely provided from higher elevations, as the water is being held under (elevation) pressure in the lower aquifer. Under pumping conditions, a flow reversal is generated around the pumping well, which could draw water levels below the confining layer. Within the areal extent of this downward flow reversal, some additional recharge to the lower aquifer could be provided by leakage through the confining layer.

Ground-water flow in the lower sand and gravel aquifer is most likely from the north and south, toward the center of the valley, and down-valley. [illegible] The shallow water table which exists in the upper (unconfined) aquifer most likely slopes gently toward the Willoughby River under normal conditions, and discharges into the river. Under high river stage conditions, flow may be temporarily reversed such that the river recharges the upper aquifer.

AQUIFER EVALUATION

Pumping Test Methodology

A step rate pumping test was conducted to assess well and aquifer characteristics, and to choose an appropriate pumping rate for the long term, constant rate test. The step test (conducted on December 9, 1988) was designed to consist of four pumping periods (steps) of 100 minutes each in duration. The pumping rate was increased at the end of each step, from 93 gallons per minute (gpm) to 201, and 303 gpm. The fourth and final step was to be run at the pump's highest achievable rate. It was discovered, however, that the pump would not deliver a consistent rate in excess of the 303 gpm used during the third step, so the test was continued at the 303 gpm rate for an additional 100 minutes. Recovery readings were made for 15 minutes following the conclusion of the final step. Due to severely cold weather conditions (approximately zero degrees Fahrenheit), only the pumping well water levels were monitored during the step test. Flowing artesian observation wells could not be kept from freezing to adequately measure water levels. Step test data are provided in Appendix C of this report.

A 72-hour constant-rate pumping test was conducted on the 8-inch diameter test well at an average rate of 302 gallons per minute (gpm) to evaluate aquifer characteristics and sustained safe yield. The test was begun on December 12, and the pump was shut off on December 15, 1988. Recovery measurements were made for 24 hours following the conclusion of the pumping period.

Discharge from both of the pumping tests was routed through a flexible 6-inch diameter hose to the Willoughby River, approximately 145 feet from the production well. Plastic sheeting was laid below the hose discharge and in a trench to dissipate the energy of the flowing water such that sediment erosion was minimized. Pumping rates were measured using orifice plates and a piezometer tube. Piezometer tube readings and corresponding flow rates are provided in Appendix C.

The pumping rate decreased during the three-day test period, from 341 gpm for the first 10 minutes of pumping, to 292 gpm at the conclusion of the test. Although a plot of flow against time would appear to indicate decreasing steps of the pumping rate, this would only be a manifestation of the accuracy to which the piezometer tube can be read in the field. While pumping, the water level in the piezometer tube rises and falls, and a measurement is made at the mid point of the fluctuations, in this case to the nearest 0.5 inch. At the test pumping rate, the drop of water level in the piezometer tube by 0.5 inch corresponds to a flow drop of 12 to 13 gpm. In reality, the flow rate probably dropped gradually, as no flow adjustments were made manually. Where appropriate, the actual approx pumping rate is used in calculations. A time-weighted average (Appendix C) of 302 gpm is used in other calculations.

Water levels in the pumping well and in 6 observation wells (Figure 2) were monitored for 72 hours of pumping, and 24 hours of recovery. A potable antifreeze was added to each of the observation wells two hours into the recovery period to facilitate subsequent measurements. This antifreeze is marketed largely for use in recreational vehicle water systems, and will not harm water quality. Water level measurements and calculated drawdowns/recoveries are also provided in Appendix C. Staff gauges were installed in the Willoughby River both upstream and downstream of the well discharge, but severely cold temperatures caused the surface of the river to freeze, and river stages could not be accurately monitored.

A rain gauge was placed approximately 20 feet from the pumping well. Daily precipitation measurements could not be accurately recorded during the pumping test, as the only precipitation that fell during this period was snow. Weather conditions remained below freezing for the duration of the pumping and recovery periods. The Burlington, Vermont National Weather Service data indicate that a total of 0.59 inches of precipitation fell in the area between December 9 and December 16, 1988 (personal communication, National Weather Service).

Aquifer and Well Characteristics

Pumping test data were evaluated using standard methods to determine aquifer and well characteristics. These characteristics can be used to determine a safe, sustained yield for a well at this site, and to delineate wellhead protection areas.

Step-Test Analysis

Bruin and Hudson Analysis. The step test was analyzed using the Bruin and Hudson (1961) method. The analysis is provided in Appendix D, and results are described below.

The well's specific capacity is observed to decrease with increasing pumping rate, from 51.5 gallons per minute per foot (gpm/ft) while pumping at 93 gpm, to 29.0 gpm/ft while pumping at 303 gpm. The analysis also shows that the well's efficiency drops with increasing pumping rate, from 66 percent at 91 gpm, to 37 percent at 303 gpm.

i.e. [illegible] test well development, [illegible] efficient [illegible]

Based on the efficiency rates calculated from the step test, the drawdown trends observed during the step test, and the capacity of the test pump, a rate of 300 gpm was chosen for the constant rate test. The rate was considered safe for the three day test period after applying conservative theoretical boundary conditions to the extension of the time-drawdown trend, and still calculating adequate available drawdown in the well for the three day pumping period.

Constant-Rate Test Analysis

Jacob Straight-Line Analyses. Semi-logarithmic plots of time versus drawdown and recovery observed in the constant rate test were graphed for Jacob Straight-Line analyses (Appendix E). The Jacob analysis provides values of the aquifer transmissivity and coefficient of storage based on recorded water levels at each observation well. Table 1 presents a summary of the aquifer

TABLE 1
AQUIFER CHARACTERISTICS
JACOB ANALYSES

Well	r (feet)	Time of slope break (mins)	Δs	t_o (mins)	T (gpd/ft)	S
PWD	0.33	240	2.8		31,000	
			4.4		18,300 boundary	
PWR		205	2.2		36,200	
			4.2		19,000	
				residual →	12,300	
OW-1D	102	250	3.2	6.2	27,100	.001
			5.2	24.5	15,200	.001
OW-1R		290	2.7	5.0	29,400	.001
			4.2	23.5	18,900	.001
OW-2D	48	190	2.7	3.8	31,900	.01
			5.2	24	15,600	.01
OW-2R		250	2.4	2.4	33,100	.01
			4.3	23	18,500	.01
OW-3D	102	110	2.6	5.0	33,400	.001
			4.8	22.5	16,900	.001
OW-3R		400	2.7	5.6	29,400	.001
			4.8	38	16,600	.01
OW-3AD	123	1300	3.2	290	24,700	.01
			7.6	690	10,200	.01
OW-5AD	9	120	2.0	1.5	43,700	.1
			5.2	22.5	15,600	.1
OW-5AR		210	2.1	2.4	39,700	.1
			4.3	25	18,500	1

T Range : 24,700 to 43,700 gpd/ft

Average T: 32,700 gpd/ft

Key:
- PW Pumping Well
- OW Observation Well
- D Drawdown Data
- R Recovery Data
- r Radial distance from pumping well
- Δs Amount of drawdown or recovery over one log cycle of time
- T Transmissivity
- t_o Time at which zero drawdown is projected
- S Coefficient of storage

characteristics derived from these analyses. Aquifer transmissivity calculated from the Jacob analyses ranges from 24,700 to 43,700 gallons per day per foot (gpd/ft). The average transmissivity value is 32,700 gpd/ft. The coefficient of storage ranges between 0.001 and 0.1. A storage coefficient between 0.001 and 0.01 is likely representative of the aquifer, as indicated by the clay confining layer observed during the test drilling, and the flowing artesian conditions at the site.

Several of the plots exhibit an increase in the rate of drawdown with time, manifested by an increase in slope (and an apparent decrease in transmissivity) after approximately 200 minutes of pumping at 302 gpm. The increased drawdown rates are apparently the result of the well's cone of influence encountering a geologic barrier which is a negative flow boundary. It is likely that the boundary observed at this early time represents the bedrock valley walls which are observed in the well vicinity. After the negative flow boundary is encountered by the well's cone of influence, the cone expands in other directions (i.e., across the floodplain) to intercept the recharge demanded by the well. If sufficient recharge cannot be intercepted by the cone of influence, the drawdown rate in the aquifer (and in the well) could increase to the point of dewatering the aquifer formation. In the case of the three day aquifer test, the drawdown rate stabilized after the first negative boundary was met, and the water level in the pumping well was drawn down by a maximum of 11.75 feet at the conclusion of the test.

The 8-inch test well's specific capacity after pumping at 302 gpm for three days is calculated to be 25.7 gpm/ft. This value is comparable to the specific capacity of 29.0 gpm/ft derived from the step-rate test. 302/11.75 : 25.7

Residual-Drawdown Analysis. A residual drawdown analysis was made using data recorded during aquifer recovery from the pumping well. This analysis gauges aquifer performance under the pumping test conditions. The residual drawdown plot is included in Appendix E, and shows that at the test pumping rate of 302 gpm, water levels very nearly rebound to the original static in the pumping well after a recovery period equal to the pumping period. Thus, for a short pumping duration, the cone of influence intercepts sufficient recharge to support a withdrawal of 302 gpm.

Distance-Drawdown Analysis. Distance-drawdown data plotted at 60 minutes, 1,440 minutes (24 hrs), and 4,290 minutes (71.5 hrs) of pumping were analyzed to determine aquifer characteristics. The distance-drawdown analysis provides an estimate of the extent of the cone of influence generated under the test pumping conditions, and also provides a measure of well efficiency. Aquifer characteristics calculated from distance-drawdown analyses generally provide representative values by integrating data from all test observation points. The distance-drawdown analyses (Appendix F) consistently yield conservative aquifer transmissivities of approximately 32,000 gpd/ft. The values are considered conservative because the analyses incorporate the drawdown levels observed in the pumping well. These are generally not included in a distance-drawdown evaluation, as drawdown in the pumping well can be significantly increased due to well efficiency. The plot of a distance-drawdown analysis which includes the drawdown at the pumping well results in a higher slope than if the pumping well were not included, and therefore results in a lower transmissivity value. The aquifer coefficient of storage calculated from the late-time analysis is 0.001.

The distance-drawdown analysis indicates that the theoretical cone of influence, after 4,290 minutes (3 days) of pumping, extends to approximately 3,200 feet. The analysis assumes homogeneous and isotropic geological conditions, and a cone which extends radially from the well. More realistically, the cone extends to the bedrock walls near the pumping well and into the river valley.

Theis Analysis. Log-log plots of time versus drawdown at two observation wells were used for Theis curve-matching analyses (Lohman, 1979). The plots and calculations are provided in Appendix G. Aquifer transmissivity values derived from these analyses are 35,300 and 25,600 gpd/ft for OW-5A and OW-2, respectively. The coefficient of storage calculated from these analyses ranges from 0.001 to 0.1. The curves matched well on the non-leaky type curve, indicating that insignificant recharge was provided through the confining layer.

Table 2 summarizes the aquifer characteristics derived from the analyses discussed above. Based on the results of the Jacob Straight-Line analyses, the distance-drawdown relationship, and the curve matching techniques, an aquifer transmissivity of 30,000 gpd/ft, and a coefficient of storage of 0.01 were chosen as representative aquifer characteristics.

TABLE 2
SUMMARY OF AQUIFER CHARACTERISTICS

Analytical Method	Transmissivity gpd/ft	Coefficient of Storage, Unitless
Jacob Straight-Line (Time Drawdown and Recovery)	24,700 to 43,700	10^{-3} to 10^{-1}
Distance-Drawdown	32,000	10^{-3} to 10^{-1}
Theis Curve-Matching	25,600 to 35,300	10^{-3} to 10^{-1}

Representative Aquifer Characteristics:

$T = 30{,}000$ gpd/ft

$S = 10^{-2}$ 0.01

Extent of River Infiltration

The effect of infiltration from the Willoughby River under pumping stresses is likely minimal due to the river's shallow depth and the existence of a clay aquitard separating it from the lower aquifer. Although no stage data could be collected from the river due to ice conditions, observation well OW-4 is located near the river (on the same side as the pumping well), and is screened in the shallow aquifer which is likely hydraulically connected to the river. If the river (and thus the shallow aquifer) were hydraulically connected with the lower aquifer near the pumping well, it is likely that OW-4 water levels would respond to pumping and recovery periods. Water level data from OW-4 (Appendix C) show no clear correlation to pumping and recovery periods. It is possible, however, that a hydraulic connection exists between the bedrock outcrop at Willoughby Falls (to the east) and the pumping well. It is likely that the clay confining layer "pinches out" or thins near the bedrock walls (Figure 4). In addition, the scouring action of the falls may have eroded the confining clay layer at this location, providing a route of entry for direct recharge into the lower aquifer.

Actual recharge from the river varies, dependent on lateral changes in streambed permeability, the river channel geometry, the head differential between the river and the lower aquifer, the permeability of the subsurface materials in contact with the river and the river water temperature. All of these factors may vary throughout the area over which the river contributes flow to the aquifer, and over time.

A reversal in vertical gradient was observed during the 72-hour test at observation wells 3A (screened in the lower aquifer) and 4 (screened in the upper aquifer). The wells are located approximately 125 feet from the 8-inch well, and are three feet apart. After approximately 2,400 minutes (40 hours) of pumping at 302 gpm, water levels in OW-3A fell below those observed in OW-4. Water levels in OW-4 did not decline in response to pumping, however. The maximum difference in water levels between the wells was on the order of three feet at the end of the pumping period. The vertical gradient observed at this location during the test ranged between an upward value of +0.06 (static conditions), and a downward value of -0.06 (at the end of the pumping period). It is likely that greater vertical gradient reversals exist within the radius of 125 feet around the pumping well, and to lesser extents outside of this radius to an unknown distance.

Downward flow could only occur where drawdown in the lower aquifer caused a reversal in the vertical gradient. The area in which this would occur would be limited by the geometry of the drawdown cone at a given time. If the water level in the lower aquifer were to fall below the confining clay layer, flow through the confining layer could be induced into the lower aquifer. No evidence of this leakage was characterized by the 72-hour test evaluation.

Safe Sustained Yield

A safe, sustained pumping rate for the test well was established using empirical data from the pumping well, and the boundary conditions observed during constant rate test. These results were then extrapolated to a safe aquifer yield, and to a proposed permanent 10 X 16-inch diameter gravel packed well.

Actual drawdown levels versus time observed in the pumping well for the 3 days of the pumping test were plotted on a semilogarithmic scale. These values indicate actual drawdown trends including a boundary observed after approximately 200 minutes of pumping at 302 gpm. To further predict drawdown levels with time, an additional boundary condition (i.e., doubled time-drawdown slope) was superimposed at the end of the pumping test period (4,290 minutes), and was extended to 180 days, to simulate 180 days of continuous pumping at 302 gpm. The resultant predicted drawdown plot is provided in Appendix H. The additional boundary condition is conservative, and simulates the effect of the well's cone of influence intercepting the bedrock walls across the Willoughby River valley. In addition, the trends observed during the 3-day pumping test were essentially without recharge to the aquifer. The extrapolation is thus conservative with respect to aquifer recharge.

The evaluation predicts that drawdown in the 8-inch well after 180 days of continuous pumping at 302 gpm would be approximately 33 feet. Further pumping of the well at twice this rate for a period of seven additional days would increase the drawdown to 35 feet. Total available drawdown in the well is approximately 55 feet. Thus, water levels would remain 20 feet above the well screen after the modeled conditions.

Safe yield for the aquifer is extrapolated from the analysis described above. As the slope of time versus drawdown is directly proportional to the pumping rate, drawdown at higher pumping rates can be easily calculated. Considering that the 8-inch diameter test well is only 37 percent efficient, and that the safe available drawdown in the aquifer is 55 feet, the extrapolation, when corrected for a well that is 74 percent efficient, reveals that 950 gpm can be withdrawn from the aquifer.

An additional method for determining safe yield, as required by the Massachusetts Safe Drinking Water Guidelines and Policies (Revised January, 1989) was applied to this site for comparison. If a permanent well was capable of achieving a specific capacity of 50 gpm/ft, the analysis indicates that the safe yield of the well would be over 2,000 gpm. This analysis does not consider any negative boundary conditions, however, and is thus considered inappropriate for this site. All calculations described above are provided in Appendix H.

Preliminary Basis of Well Design:

It is likely that the permanent well screen will be limiting to the withdrawal rate achieved in this aquifer. The proposed 10 X 16-inch diameter gravel packed well, if constructed with a 10 foot long, 90-slot screen would be capable of providing approximately 540 gpm at the maximum screen entrance velocity conditions.

GROUND-WATER QUALITY

In order to assess the present and future quality of water available from the production well, Wright Engineering inventoried potential sources of contamination within a 2,000 foot radius of the test well, and installed three monitoring wells near underground storage tanks at local residences. Water samples from the 8-inch well were collected by Wright Engineering during the 72-hour pumping test for laboratory analysis. Based upon the hydrogeologic setting and the identified potential sources of contamination, preliminary well-head protection areas (WHPA's) are identified.

Potential Sources of Contamination

A survey was conducted by Wright Engineering to identify potential sources of contamination within a 2,000 foot radius of the test well location. The majority of the area is sparsely populated and residential. As the proposed well site is located at a topographically low point (in the Willoughby River floodplain), approximately half of the 2,000 foot radial area is considered to be hydraulically upgradient of the site.

The survey identified several underground storage tanks in this area, and an existing and former location of electrical sub-stations. In addition, the municipal garage is located approximately 900 feet to the west of the Tarbox site. Salt storage and/or underground storage tanks at the garage may be potential sources of ground-water contamination, although Ground

Water Associates has not determined whether these potential sources exist at this location. Several manholes, catchbasins, and drains were identified along Church Street, which runs adjacent to the Tarbox meadow on the west side. These may serve as potential sources of contamination depending upon their structural integrity and the contributing sources which feed them.

Two gasoline stations are located on the western edge of the 2,000-foot radial area. An Ethan-Allen plant also lies on the western boundary of the area investigated. This Ethan-Allen facility is registered as a large-quantity generator under the U.S. EPA Resource Conservation and Recovery Act (RCRA). According to Vermont Department of Natural Resources officials, the site is currently undergoing a ground water remediation program (personal communication). The gas stations and the Ethan-Allen plant lie in the Barton River drainage, and therefore are unlikely to impact ground-water quality at the Tarbox site.

Three 2.5-inch diameter monitoring wells were installed near residential underground storage tanks located within 1,100 feet of the 8-inch diameter test well. The drilling logs for these wells are provided in Appendix A. Monitoring well 5 is located on Church Street, approximately 600 feet from the test well, and near a 250 gallon underground gasoline tank. Monitoring well 6 is located at the intersection of Union and School Streets, approximately 1,100 feet from the test well, and near an underground fuel tank of unknown capacity. Monitoring well 7 is located on East Street (approximately 800 feet from the test well), near an underground 250 gallon gasoline tank. No odors or soil stains were observed by the driller during monitoring well installation. A perched water table was observed at each of the borings.

According to Wright Engineering, the Village of Orleans is entirely sewered. Sewering is generally considered to be a preferable alternative to septic systems with regards to aquifer protection, but structural deficiencies resulting in exfiltrating sewers may adversely affect ground-water quality. Septic systems were noted to exist across the Willoughby River. Leaching fields are likely set in relatively shallow overburden deposits. If the septic systems are properly designed, constructed, and maintained, they should not pose a threat to ground water quality.

Canadian Pacific railroad tracks cross through a portion of the surveyed area, along the Barton River. Train tracks present contamination concerns due to defoliants used on railroad beds, and the potential for a spill of transported material.

The potential sources of contamination described above have only been identified and located within a 2,000 foot radius of the test well. No assessment has been made regarding the degree of contamination which may currently exist, or has existed in the past. and [illegible] report shows minimal [illegible]

Laboratory Analysis

Samples of the 8-inch test well water were collected near the end of the pumping period of the aquifer test. The samples were analyzed for inorganic primary and secondary public water supply standards, volatile organic compounds, and radioactive levels. Analyses were conducted by the Vermont Department of Health Laboratory. Laboratory results are provided in Appendix I.

All inorganic compounds tested for are within the Maximum Contaminant Level (MCL) or the Maximum Contaminant Level Goal (MCLG) required by the state. The water is considered to be hard, based on the hardness value of 231 mg/L (as calcium carbonate) determined by the state laboratory. The federal secondary standard for hardness is 150 mg/L. Nitrates were not analyzed, as the holding time for the sample exceeded the analytical method's allowable limit.

Volatile organic analytical results show that none of the 29 compounds tested for were detected above the minimum analytical detection limit (on the order of parts per billion).

The gross alpha evaluation indicates that the sample concentration falls below federally acceptable levels of radioactivity.

In summary, the ground water at this site meets applicable State and Federal drinking water standards for the parameters tested. Nitrates have not been assessed in this analysis. The laboratory results indicate that the water should not prove to be corrosive, nor should it cause incrustation problems. Should the pH and/or hardness levels in the future rise above 7.5 and 300 mg/L respectively, concerns of calcium carbonate incrustation may be warranted (Driscoll, 1986).

Wellhead Protection

Wellhead protection should focus on protecting aquifer recharge zones from sources of ground-water contamination. Potential sources of ground water contamination in the well site vicinity are discussed above. The hydrogeology of the drainage basin determines the location of recharge zones which should be protected from contamination sources.

The hydrogeologic setting offers some natural protection from potential contamination originating in the upper aquifer. The silt and clay layer overlying the lower aquifer in the floodplain and also noted to be present near the surface of the valley above the floodplain has a low hydraulic conductivity, and greatly slows the rate of ground-water flow. Where this aquitard exists, it would impede the flow of ground water and potential contaminants. Some of the Tarbox site test wells screened in the lower aquifer exhibit flowing artesian conditions at the ground surface. This implies that an upward flow gradient exists in these locations, and may be present in other parts of the Willoughby River Valley. The upward flow gradient would also likely impede contaminant migration to the lower aquifer. This upward gradient condition may be reversed in areas adjacent to the pumping well under pumping conditions. During the 72-hour aquifer test conducted at a pumping rate of 302 gpm, the vertical gradient was observed to reverse after 40 hours at a distance of 125 feet from the pumping well. The lateral extent of the area in which flow reversal occurred is unknown, and is also dependent upon the composition of subsurface materials in the vicinity. Even where vertical gradients are downward, the silt and clay layer would greatly impede ground water and contaminant migration to the lower aquifer.

The Vermont Department of Public Health Water Supply Standards (1988) prescribe that a well isolation zone and wellhead protection areas be designated surrounding all new ground water sources. Suggested areas for aquifer protection are discussed below.

The State requires that a well isolation zone be designated, in which only limited land uses are permitted. Documented control of the well site isolation zone must be demonstrated to the Vermont Department of Public Health. The State guidelines suggest that a distance of 200 feet be designated as the well isolation zone. This area is within the boundary of the Tarbox site. Permitted and non-permitted land uses in the well isolation zone are specified by the State as follows.

1. Land uses permitted within the well isolation zones include:
   a) playgrounds, ballfields, tennis courts
   b) seasonal light-duty roads

c) conservation zones

d) controlled use of fertilizers

e) other uses may be allowed provided they have the approval of the Division.

2. Land uses not permitted within the well isolation zone include:

   a) application of pesticides and herbicides

   b) buildings other than those required for the water system.

   c) parking of motor vehicles

   d) chemical storage other than that required by the water system

   e) swimming pools

   f) salted or paved roads passing through the area

   g) leachfields, septic tanks, and sewer lines

   h) any other activity which may contaminate the water supply.

Primary and secondary WHPA's are to be designated, which are the land surface areas where land use activity will (may) or can "unreasonably" affect ground-water quality. The WHPA is meant to be used in ground-water management to protect both water quality and quantity. The State suggests use of two-year and five-year time-of-travel zones (the area in which ground water water is expected to reach the pumping well within two and five years) to define primary and secondary WHPA's, respectively. The two-year time-of-travel zone (primary WHPA) is to be used as the area within which septic system larger than single family home systems per ANR DEC EPR's siting will be evaluated. The five-year time-of-travel area (secondary WHPA) is to be used to review other proposed activities.

Time of travel evaluations are not easily applied to the hydrogeologic conditions encountered at the Tarbox site in Orleans, and with the site-specific data collected during this study. The lateral extent of the aquifer formation is known to be limited by bedrock valley walls in the well site vicinity. It is likely that the aquifer extends across the Willoughby River floodplain, but the continuous extent of the formation on the north side of the Willoughby River and the hydraulic gradients within the aquifer remain unconfirmed. In addition, the lateral and vertical extent of the clay confining layer beyond the Willoughby River is also unknown. Without this hydrogeologic data, long-term extrapolations of ground-water flow paths cannot be reasonably made.

Recharge zones to the lower aquifer were not assessed during this study. Given the confined aquifer and artesian flow conditions at this site, it is likely that the lower aquifer derives recharge from higher elevations, which may be local bedrock exposures. In this scenario, recharge would flow through bedrock fractures, and into the lower aquifer, where it is held under pressure. An evaluation of the potential hydraulic connection between the bedrock and the lower aquifer was beyond the scope of this study, however.

As limited data are available to assess aquifer recharge zones and ground-water flow paths in the well's area, an infiltration model was utilized to delineate the preliminary secondary wellhead protection area. The surficial geologic map of Vermont (Doll, 1970) indicates that the drainage area for the well site (Appendix B) is covered largely by till. A conservative infiltration rate of 0.58 feet annually was applied to balance a discharge rate of 540 gpm (the likely maximum production well withdrawal rate). A radial distance of 4,560 feet was derived to describe the area required to provide a recharge rate of 540 gpm to the aquifer. This area, shown in Figure 5, is centered around the pumping well, and may be more realistically oriented upgradient of the well. Some flow to the well is likely to be induced from the down-valley extent of the aquifer, although the hydrogeology of this portion of the valley is not well enough understood to quantitatively determine the distance to which the induced flow may exist. Thus, the preliminary secondary WHPA is the radial distance of 4,560 feet from the production well.

The primary WHPA was determined using a predicted distance-drawdown analysis which simulates drawdown conditions at a withdrawal rate of 540 gpm (the estimated maximum yield of a permanent gravel-packed well) after 180 days of continuous pumping, followed by seven days of pumping at twice this rate (the safe yield conditions). One foot of drawdown is predicted to occur at a distance of 1,900 feet from the pumping well. Although this analysis assumes infinite, homogeneous aquifer conditions which do not exist at this site, it incorporates effective aquifer characteristics. Thus, a preliminary primary

based on Q = 540 gpm!

WHPA radial distance of 1,900 feet is proposed. Septic systems should not be allowed in areas where sewering is accessible. For other areas of the village where the sewerage system may not be accessible (e.g., on the north side of the Willoughby River), septic system siting within 1,900 feet of the well site should be reviewed. Test pits should be required in these areas to determine the suitability of subsurface materials for leaching fields. Standard septic system design criteria should be used, and plans should be reviewed by a qualified engineer representing the Village or Town.

Figure 5 shows the extent of the proposed well site isolation zone and the preliminary well head protection areas.

The Village of Orleans should determine appropriate zoning and/or land use restrictions within the secondary WHPA to assure ground-water quality. Measures which could be taken to protect against ground-water contamination include, but are not limited to:

- restricted industrial/manufacturing development,
- underground storage tank restrictions or construction regulations,
- waste disposal restrictions,
- decreased road salt application,
- maintenance of the inventory of potential sources of contamination in the WHPA,
- proper maintenance and inspection of the existing sewer

Figure 5. Preliminary well site isolation and well head protection areas.
Circles indicate, from inner to outer, the 200-foot radial well site isolation zone, the 1900-(based on ...) foot radial primary WHPA, and the 4560-foot (based on infiltration) radial secondary WHPA. Basemap adapted from the Memphremagog Quadrangle U. S. Geological Survey 15 minute series topographic map, 1953.

QUADRANGLE LOCATION

~3200' at Q = 300 based on infiltration

0 5280 feet

system, and engineering oversight of future sewer expansion,

- herbicide and pesticide use restrictions, and
- ground-water quality monitoring.

These measures can be implemented through the development of zoning by-laws which contain specific language to protect aquifer recharge areas.

In order to assure an adequate ground-water supply, development of additional ground-water resources in the Willoughby River valley should be carefully assessed such that interference between the Tarbox well and a future well is minimized, while maximizing total yield.

CONCLUSIONS AND RECOMMENDATIONS

Review and evaluation of available hydrogeologic data, pumping tests conducted on the 8-inch diameter test well, and laboratory chemical analyses yield the following conclusions and recommendations:

1. The aquifer material within the vicinity of the pumping well consists of sand and gravel outwash deposits confined by a glaciolacustrine silt and clay layer. Flowing artesian conditions exist at the ground surface at the Tarbox site, indicating a natural upward vertical hydraulic gradient. Impermeable geologic boundaries limit the lateral extent of the aquifer to the east, south, and west.

2. The drainage area contributing to the well location encompasses approximately 55 square miles. Based on conservative rainfall infiltration rates, this equates to a total basin recharge potential of approximately 11,000 gpm.

3. Pumping test data from the long-term constant-rate test indicate the aquifer has a representative transmissivity of 30,000 gpd/ft, and a storage coefficient of 0.01. Test well efficiency at a pumping rate of 302 gpm is approximately 37 percent. The specific capacity of the test well is approximately 26 gpm/ft.

4. A safe aquifer yield of 950 gpm was determined based on the extension of empirical pumping test data with the effects of two negative boundaries. More realistically, using a well efficiency of 74 percent and at maximum screen entrance velocity, a permanent gravel-packed well at this site will provide a sustained yield of 540 gpm.

5. Following construction of a permanent well, a step-rate test followed by a short-term constant-rate test should be conducted to assess the well's efficiency such that a more accurate well yield can be determined. Monthly monitoring of ground-water levels is suggested to allow the design of a proper maintenance and redevelopment schedule. Weekly monitoring of water levels should be made during the first quarter of well operation to confirm well performance.

6. Potential sources of contamination within a 2,000 foot radius of the well site were identified by Wright Engineering. No assessment has been made regarding the current or past extent of contamination at these locations.

7. Preliminary well head protection areas (WHPA's) surrounding the well site have been determined as required by the Vermont Department of Health. A well site isolation zone extends to a radial distance of 200 feet around the well; the primary WHPA consists of the

1,900 foot radial distance from the well; and the secondary WHPA extends to a radial distance of 4,560 feet from the well site. Land use restrictions in the well site isolation zone are specified by the State. The Village of Orleans should institute controls to protect ground-water resources in the vicinity of the Tarbox site in the Willoughby River Valley.

8. Ground water at the production well is generally of good quality, and meets State and Federal drinking water standards for those parameters tested. Hardness and pH levels may cause encrustation problems in a permanent well, and necessitate a regular well maintenance schedule. Nitrates should be retested.

REFERENCES

Bruin, J. and H. E. Hudson 1961. Selected Methods for Pumping Test Analysis, Report of Investigation 25, State of Illinois Water Survey Division, Urbana, Illinois.

Doll, C. G. 1951. Geology of the Memphremagog Quadrangle and the Southeastern Portion of the Irasburg Quadrangle, Vermont. Bulletin No. 3, Vermont Geological Survey, Montpelier, Vermont.

Doll, C. G. 1970. Surficial Geologic Map of Vermont, State of Vermont.

Driscoll, Fletcher. 1986. Groundwater and Wells. Johnson Division, St. Paul, Minnesota.

Freeze, A., and J. Cherry. 1979. Groundwater. Prentice-Hall, Inc. Englewood Cliffs, New Jersey.

Lohman, S. W. 1979. Ground-Water Hydraulics. Geologic Survey Professional Paper 708, U. S. Government Printing Office, Washington, D. C.

Walton, William C. 1970. Selected Analytical Methods for Well and Aquifer Evaluation. Bulletin 49, State of Illinois Water Survey Division, Urbana, Illinois.

Walton, William C. 1970. Groundwater Resource Evaluation. McGraw-Hill, Inc., New York, N.Y.

Wright Engineering, LTD. Water System Improvements for the Village of Orleans, Vermont, Drawing Location Plan, Engineer's Job. No. 8821.

Wright Engineering, LTD. Contamination Survey for Proposed Well Site, Base Map: Village of Orleans, Town of Barton, Vermont Existing Water System, Engineer's Job No. 8737.

LIMITATIONS OF WORK

The Evaluation of the Tarbox well site was performed for the exclusive use of Wright Engineering, LTD. The conclusions drawn by Ground Water Associates, Inc., are based solely on information gathered to date. Information that may be gathered in the future may modify certain aspects of the conclusions reported herein. This report has been prepared in accordance with generally accepted hydrogeological and geophysical practices. No other warranty, expressed or implied, is made.